Rapport sur les causes techniques de l'effondrement du viaduc de la Concorde

Annexe A7
Analyse de la conception des viaducs de la Concorde et de Blois

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Annexe A7

EVALUATION OF STRUCTURAL DESIGN - DE LA CONCORDE AND DE BLOIS BRIDGES

Report Prepared for:

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Table of Contents

EVALUATION OF STRUCTURAL DESIGN - DE LA CONCORDE AND DE BLOIS BRIDGES

The main purpose of this design evaluation is to determine if the original design, as shown on the structural drawings, conforms to the requirements of the Canadian Standards Association Standard CSA S6-1966 – Design of Highway Bridges and the CSA S6-2006 Canadian Highway Bridge Design Code.

This report presents detailed calculations of design loads for both editions of the S6 Code as determined by hand methods (Chapters 1 and 2) and these design loads are then compared (Chapter 3). The design and drawings are then evaluated in accordance with the requirements of the 1966 Code (Chapter 4).

The results from a three-dimensional (3-D) finite element analysis are presented and compared to the hand calculations (Chapter 5). The design is evaluated in accordance with the requirements of the 2006 Code (Chapter 6).

Some of the key aspects of the design requirements are reviewed, namely the requirements for shear (Chapter 7) and the requirements for 'disturbed' regions (Chapter 8).

The designs and Code compliance of the Concorde overpass and de Blois overpass are compared (Chapter 9).

1. Evaluation of Structural Design Loads Using CSA S6-1966

1.1 Dead Loads on Cantilever at Beam Seat

The dead load on the neoprene pads is caused by the weight of the precast, pretensioned hollow box girders, the sidewalks, the concrete topping, the asphalt wearing surface and the railings.

(a) Weight of Hollow Box Girder

Each hollow box girder has outside dimensions of 4'-0" wide by 3'-6" deep. The girder has two webs, each 5 in. thick, a top flange of 5.5 in., a bottom flange of 5.5 in., 3 in. chamfers on the inner corners, three 8 in. thick intermediate diaphragms girder (includes 3 in. chamfers). The total length of each girder is 91'-5", with a skew angle of 69.5º. There is a solid portion with a dap at each end.

The cross-sectional area of the hollow portion is:

$$A = \frac{(48\times 42)-(38\times 31)+2(3\times 3)}{144} = 5.944\ \text{ft}^2$$

The volume of one diaphragm $= 8\times \frac{38}{\sin 69.6^\circ}\frac{31}{12^3} = 5.822\ \text{ft}^3$

The volume of one end block $= 24\times \frac{48}{\sin 69.5^\circ}\frac{42}{12^3} + 18\times \frac{48}{\sin 69.5^\circ}\frac{21}{12^3} = 41.103\ \text{ft}^3$

The total volume of concrete in one girder is:

$$5.944\left(91.4167 - \frac{2\times 42}{\sin 69.5^\circ \times 12}\right) + 3\times 5.822 + 2\times 41.104 = 598.67\ \text{ft}^3$$

The weight of concrete in one girder $= 0.150\times 598.67 = 89.80\ \text{kips}$

The encapsulated formwork inside each girder (5/16 in. plywood plus 1 x 4 in. framing at 16 in.) that was used to form the void has a weight of about 20 plf. This results in a total load in each girder $= 91.4\times 20/1000 = 1.828\ \text{kips}$

The total weight of one girder $= 89.80 + 1.83 = 91.63\ \text{kips}$

(b) Weight of Cast-in-Place Topping plus Median plus Sidewalks

Total topping weight (3.5 in. thick) per half width of bridge, supported by 10 girders

$$= \left(35.5\times 91.4167\times \frac{3.5}{12}\right)\times 0.150 = 141.98\ \text{kips}$$

Total weight of asphalt topping (2.5 in. thick) per half width of bridge

$$= \left(35.5\times 91.4167\times \frac{2.5}{12}\right)\times 0.150 = 101.42\ \text{kips}$$

Weight of one-half of central median $= \left(2.5\times \frac{15.5}{12}\times 91.4167\right)\times 0.150 = 44.28\ \text{kips}$

Total weight of sidewalk per half width of bridge

$$= \left[\left(7\times \frac{16}{12}\right) - \left(\frac{8\times 48}{2\times 144}\right)\right]\times 91.4167\times 0.150 = 109.7\ \text{kips}$$

(c) Weight of Guard Rails

Total weight of guard rail per half width of bridge at 30 plf approx

$= 30\times 91.4167/1000 = 2.74\ \text{kips}$

(d) Average dead load reaction on one neoprene pad

$$=\frac{91.63}{2}+\left(\frac{141.98+101.42+44.28+109.7+2.74}{2\times 10}\right)=65.82\,\text{kips}$$

1.2 Live Loads on Cantilever at Beam Seat

An H20-S16 standard truck loading has been assumed in accordance with the requirements of CSA Standard S6-1966.
The width of the design traffic lane (Clause 5.1.6.1) for a roadway width, W_c of 35.5 ft is
$W=\frac{W_c}{N}=\frac{35.5}{3}=11.833\,\text{ft}$
The span centre-to-centre of neoprene pads $=90.0\,\text{ft}$
From Table A2, for an H20-S16 loading on a simple span, the end reaction for one lane load $=64.5\,\text{kips}$. It is noted that the reaction is governed by the standard truck loading rather than the standard lane loading.

In accordance with Clause 5.2.1.1, the wheel located near the support contributes 100% of its load to the reaction. Considering the transverse distribution of wheel loads (Clause 5.2.1.2), the distribution factor depends on the girder spacing, S and hence $=\frac{S}{7.0}=\frac{4.0}{7.0}=0.5714$.

A standard H20-S16 truck has a transverse spacing between wheel loads of 6 ft. Therefore, only one line of wheels would contribute to the loading on one girder. The wheel loads are 16 kips, 16 kips and 4 kips. The critical spacing between the three axles is 14 ft.

By taking moments about one end of the 90 ft span, the end reaction due to truck loading on one neoprene pad
$$=\frac{1.0\times 90\times 16+0.5714(90-14)\times 16+0.5714(90-28)\times 4}{90}=25.3\,\text{kips}$$
Reaction including impact factor of 1.3 (Clause 5.1.11.1.c.iv) $=1.3\times 25.3=32.9\,\text{kips}$

1.3 Design Vertical Reaction at Beam Seat

The design vertical reaction on one neoprene pad $=65.82+32.9=98.7\,\text{kips}$

1.4 Design Loading on Cantilever

1.4.1 Dead Loads

The dead load on each pad from the dead load of the hollow-box girders is 65.82 kips.

The cantilever of the east abutment has a depth of 45.5 in. near the neoprene pad and a depth of $45.5+5.375=50.875$ in. at the wall support. The additional dead load from the self –weight of the cantilever can be approximated by assuming an average depth of $(45.5+50.875)/2=48.2$ in. The equivalent uniform load over a 4 ft width is:
$(4.0\times 48.2/12)\times 0.150=2.41$ kips/ft.

The loading from asphalt topping (2.5 in. thick) on a 4 ft width of the cantilever is:
$$=\left(4.0\times\frac{2.5}{12}\right)\times 0.150=0.125\ \text{kips/ft}$$

Weight of one-half of central median on the cantilever:
$=\left(2.5\times\frac{15.5}{12}\right)\times 0.150=0.484$ kips/ft to be distributed over a width of 41'-0".

Weight of sidewalk on cantilever per half width of bridge:
$=\left[\left(7\times\frac{16}{12}\right)-\left(\frac{8\times 48}{2\times 144}\right)\right]\times 0.150=1.20$ kips/ft to be distributed over a width of 41'-0".

Weight of guard rail on cantilever per half width of bridge is estimated as 30 plf.

The total equivalent dead load per unit length for a 4 ft width of cantilever can be estimated as:
$$2.41+0.125+(0.484+1.20+0.030)\times\frac{4.0}{41.0}=2.70\ \text{kips/ft}$$

Therefore, in a 4ft width of the cantilever, the maximum dead load shear at the wall support is:

$$65.82+2.70\times 13.0=100.92\ \text{kips}$$

In a 4 ft width of the cantilever the dead load moment at the face of the wall support is:

$$65.82\times 13.0+\frac{2.70\times 13^2}{2}=1084\ \text{ft kips}$$

1.4.2 Live Loads

(a) Shear

From Section 1.2 the H20-S16 truck loading plus impact is 32.9 kips.

(b) Moment

The moment from H20-S16 truck loading and impact at the face of the wall support is:

$32.9 \times 13.0 = 427.7 \text{ ft kips}$

1.4.3 Dead Load plus Live load and Impact

Total shear from dead load plus live and impact loading on a 4 ft width
$= 100.92 + 32.9 = 133.8 \text{ kips}$

Total moment from dead load plus live load and impact loading on a 4 ft width is:
$= 1084 + 427.7 = 1512 \text{ ft kips}$

2. Evaluation of Structural Design Loads Using CSA S6-2006

It is noted that the CSA S6-2006 Standard uses factored loads and factored resistances for design.

2.1 Factored Dead Loads on Cantilever at Beam Seat

The unit material weights given in Clause 3.6 and the load factors from Table 3.3 of CSA S6-2006 are:

Material	**Unit weight, kN/m^3**	**Load factor**
Bituminous wearing surface	23.5	1.50
Prestressed concrete	24.5	1.10
Reinforced concrete - Cast-in-place	24.0	1.20
Wood and all non structural components		1.20

Using these unit weights, the unfactored dead loads causing a reaction on one neoprene pad are:

Girder prestressed concrete: $\dfrac{598.67 \times 0.3048^3 \times 24.5}{2} = 207.7\,\text{kN} = 46.69\,\text{kips}$

Girder encapsulated wooden forms at approx 20 plf:

$$= \frac{20 \times 91.4167}{2 \times 1000} \times 4.4482 = 4.07\,\text{kN} = 0.91\,\text{kips}$$

Cast-in-place concrete topping weight (3.5 in. thick), supported by 10 girders

$$= \frac{\left(35.5 \times 91.4167 \times \dfrac{3.5}{12}\right) \times 0.3048^3 \times 24.0}{2 \times 10} = 32.2\,\text{kN} = 7.23\,\text{kips}$$

Asphalt topping (2.5 in. thick)

$$= \frac{\left(35.5 \times 91.4167 \times \dfrac{2.5}{12}\right) \times 0.3048^3 \times 23.5}{2 \times 10} = 22.5\,\text{kN} = 5.06\,\text{kips}$$

Weight of one-half of central median supported by 10 girders

$$= \frac{\left(2.5 \times \dfrac{15.5}{12} \times 91.4167\right) \times 0.3048^3 \times 24.0}{2 \times 10} = 10.03\,\text{kN} = 2.26\,\text{kips}$$

Total weight of sidewalk per half width of bridge

$$= \frac{\left[\left(7 \times \dfrac{16}{12}\right) - \left(\dfrac{8 \times 48}{2 \times 144}\right)\right] \times 91.4167 \times 0.3048^3 \times 24.0}{2 \times 10} = 24.85\,\text{kN} = 5.59\,\text{kips}$$

Weight of Guard Rails

Total weight of guard rail per half width of bridge at 30 plf approx

$$= \frac{30 \times 91.4167}{2 \times 10 \times 1000} \times 4.4482 = 0.61\text{kN} = 0.14\,\text{kips}$$

Total unfactored dead load reaction on one neoprene pad:

$$207.7 + 4.1 + 32.2 + 22.5 + 10.0 + 24.9 + 0.6 = 302.0\,\text{kN} = 67.9\,\text{kips}$$

2.2 Factored Live Loads on Cantilever at Beam Seat

2.2.1 Simplified Method

a) Skew

Based on the dimensions of the central spans, where the width of each independent half is taken as the outer dimensions of 10 adjacent box girders, the skew parameter is computed as 40 tan (20.5deg)//90 = 0.167. This does not exceed the limit of 1/6, which satisfies one of the conditions for use of the simplified method, Clause 5 7.1.1. As for dead loads, there is no limitation due to the skew since this is unshored construction, Clause 5.6.1.1.

b) Bridge Type

Each individual hollow-box girders has a thickness of the web defined by adjacent voids of $5\,\text{in} = 127\,\text{mm}$. With two girder side-by-side, the web thickness is 10 in = 254 mm.

This is more than 20% of the total depth $= 0.20 \times (42 + 3.5) = 9.1\,\text{in} = 231\,\text{mm}$ and hence meets the requirement for voided slabs. The depth of the void is 31 in. which does not exceed 80% of the depth $= 0.8 \times 45.5 = 36.4\,\text{in}$ and hence meets the requirement for voided slabs. In addition, full length shear keys, a composite deck slab and transverse post-tensioning are provided in 3 intermediate diaphragms. Hence, in accordance with Clauses 5.5.2, the suspended span may be treated as a "voided slab".

c) Longitudinal Moment

Width $= B = 45\,\text{ft} = 13.72\,\text{m}$
Effective width $= B_e = 40\,\text{ft} = 12.19\,\text{m}$.
$L = 90\,\text{ft} = 27.43\,\text{m}$
Deck width, W_c , is 35.5 ft = 10.82 m.

Number of design lanes, n, is 3.

Width of design lane (Clause 3.8.2), $W_e = \dfrac{W_c}{n} = \dfrac{10.82}{3} = 3.607\,\text{m}$

Factor $\mu = \dfrac{W_e - 3.3}{0.6} = \dfrac{3.607 - 3.3}{0.6} = 0.5113 \le 1.0$

From Clause 5.7.1.2.1.2 and Table 5.3 of CSA S6-2006, for Type B, Class A, and 3 design lanes:

$$F = 10.80 - \frac{8}{L} = 10.80 - \frac{8}{27.43} = 10.51\,\text{m}$$

$$C_f = 16 - \frac{30}{L} = 16 - \frac{30}{27.43} = 14.91$$

Amplification factor, $F_m = \dfrac{B}{F\left\{1+\dfrac{\mu C_f}{100}\right\}} = \dfrac{13.72}{10.51\left\{1+\dfrac{0.5113\times 14.91}{100}\right\}} = 1.213 \geq 1.05$

The maximum moment, obtained by placing the truck such that the midpoint between the centre of gravity of the design truck and the closest axle is located at the centre of the 90-foot span is 2634 kN-m, under the 175 kN axle. Therefore, the maximum moment due to the truck including DLA (Clause 3.8.4.5.3):

$M_T = 2634 \times (1+0.25) = 3293\,\text{kNm}$

R_L = modification factor for multi-lane loading = 0.8 for 3 lanes loaded (Clause 3.8.4.2)

Average longitudinal moment per metre of width, for live load:

$m_{avg} = \dfrac{nM_T R_L}{B_e} = \dfrac{3\times 3293\times 0.8}{12.19} = 648.33\,\text{kNm/m}$

Maximum longitudinal moment per metre of width, for live load:

$m = F_m m_{avg} = 1.213\times 648.33 = 786.4\,\text{kNm/m}$

d) Longitudinal Shear (Clause 5.7.1.4)

B is the total width of the bridge = 13.72 m

The width dimension that characterizes load distribution is obtained from Table 5.7.

$F = 8.40$

F reduction factor as spacing, S is less than 2.00 m $= \left(\dfrac{S}{2}\right)^{0.25} = \left(\dfrac{1.219}{2}\right)^{0.25} = 0.8836$

Amplification factor $F_v = \dfrac{B}{F} = \dfrac{13.72}{8.40\times 0.8836} = 1.848 \geq 1.05$

The critical truck location is with the truck going forward, with its second axle located over the bearing and the following 3 axles on the box girder. The maximum shear at the bearing is 440.4 kN. Therefore, the maximum shear per lane including DLA is:

$V_T = 440.4\times(1+0.25) = 550.5\,\text{kN}$.

The average longitudinal vertical shear per unit width $= v_{avg} = \dfrac{nV_T R_L}{B_e} = \dfrac{3\times 550.5\times 0.8}{12.19} = 108.4\,\text{kN/m}$

and the longitudinal vertical shear per unit width,

$$v = F_V v_{avg} = 1.848 \times 108.4 = 200.3\,\text{kN/m}$$

The shear for a 4 ft (1.219 m) width is $= 1.219 \times 200.3 = 244.2\,\text{kN} = 54.9\,\text{kips}$

2.3 Design Loads

2.3.1 Serviceability Limit States

The Serviceability Limit States (SLS) loading on a neoprene pad

$$SLS = 1.0D + 0.9L = 1.0 \times 302.0 + 0.9 \times 244.2 = 521.8\,\text{kN} = 117.3\,\text{kips}$$

2.3.2 Fatigue Limit States

For the Fatigue Limits States (FLS), from Clause 5.7.1.4.2 only a single truck with one lane loaded is considered. The value of F obtained from Table 5.8 is 3.80 for $n = 1$ and $R_L = 1$. Hence

$$F_v = \frac{B}{F} = \frac{13.72}{3.80} = 3.61 \;\; > 1.05$$

$$v_{avg} = \frac{nV_T R_L}{B_e} = \frac{1 \times 550.5 \times 1}{12.19} = 45.2$$

$$v = F_v v_{avg} = 3.61 \times 45.2 = 163.0$$

Hence the shear force on a 4 ft width $= 1.219 \times 163.0 = 198.7\,\text{kN} = 44.7\,\text{kips}$.

The Fatigue Limit States (FLS) loading on a neoprene pad is

$$FLS = 1.0D + 1.0L = 1.0 \times 302.0 + 1.0 \times 198.7 = 500.7\,\text{kN} = 112.6\,\text{kips}$$

2.3.3 Ultimate Limit States

The Ultimate Limit States (ULS) loading (ULS combination 1) on a neoprene pad

$$ULS = \alpha_D D + 1.7L$$
$$= 1.10 \times 207.7 + 1.20 \times (4.07 + 32.2 + 10.03 + 24.85 + 0.61) + 1.50 \times 22.5 + 1.7 \times 244.2$$
$$= 763.5\,\text{kN} = 171.6\,\text{kips}$$

3. Comparison of Design Loads

Table 3.1 compares the total truck weights in the CSA Standard since 1966.

Table 3.1 – Truck Weights

	Truck Designation	Truck Weight (kN)	Truck Weight (kips)
S6-1966	H20-S16	320 kN	72 kips
S6-1988	QS-660	660 kN	148.4 kips
S6-2000 & S6-2006	CL-625	625 kN	140.5 kips

Table 3.2 compares the calculated reaction on one neoprene pad using the different CSA Standards.

Table 3.2 – Reactions on neoprene pad for different standards and different loading cases

	Reaction on pad (kN)	Reaction on pad (kips)
S6-1966 (unfactored loads)	439 kN	98.7 kips
S6-2006 (SLS)	521.8 kN	117.3 kips
S6-2006 (FLS)	500.7 kN	112.6 kips
S6-2006 (ULS)	763.5 kN	171.6 kips

4. Evaluation of Drawings and Design of the Cantilever in Accordance with CSA S6-1966

The key aspects of the structural drawings, general characteristics of the bridge and the design of the cantilever are examined in accordance with the CSA Standard S6 1966.

4.1 Comments on Drawings

The structural drawings by Desjardins & Sauriol (COM-19) were stamped “Tel que construit” or “as-built” but did not reflect a number of subsequent changes as shown on the drawings, with revisions dated August 17, 1970. The revised drawings showed the following important changes:

- The details of the pins at the beam seat on the west end were indicated (see Fig. 4.1 and 4.2)

- the 2 ½ in. thickening of the cantilever at the east expansion joint was indicated (see Fig. 4.1 and 4.2)
- the total length of the 2 in. by 3/8 in. thick steel plates that were welded to the steel angle at the edge of the expansion joint were changed from 18 in. to 12 in. Accounting for the end anchorages of these plates resulted in an embedment length that was reduced from 12 in. to 9 in.
- the lengths of the #14 main longitudinal reinforcement was changed. This change resulted in #14 bars at a spacing of 6 in. in the cantilever, rather than 12 in.

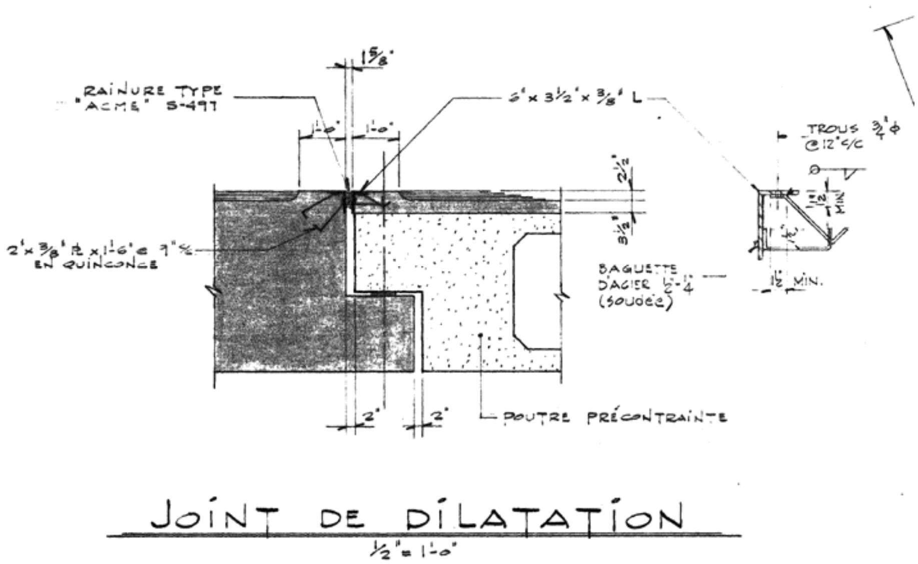

Figure 4.1 – Details of pins and expansion joint reinforcement before revisions (COM-19, p. 8)

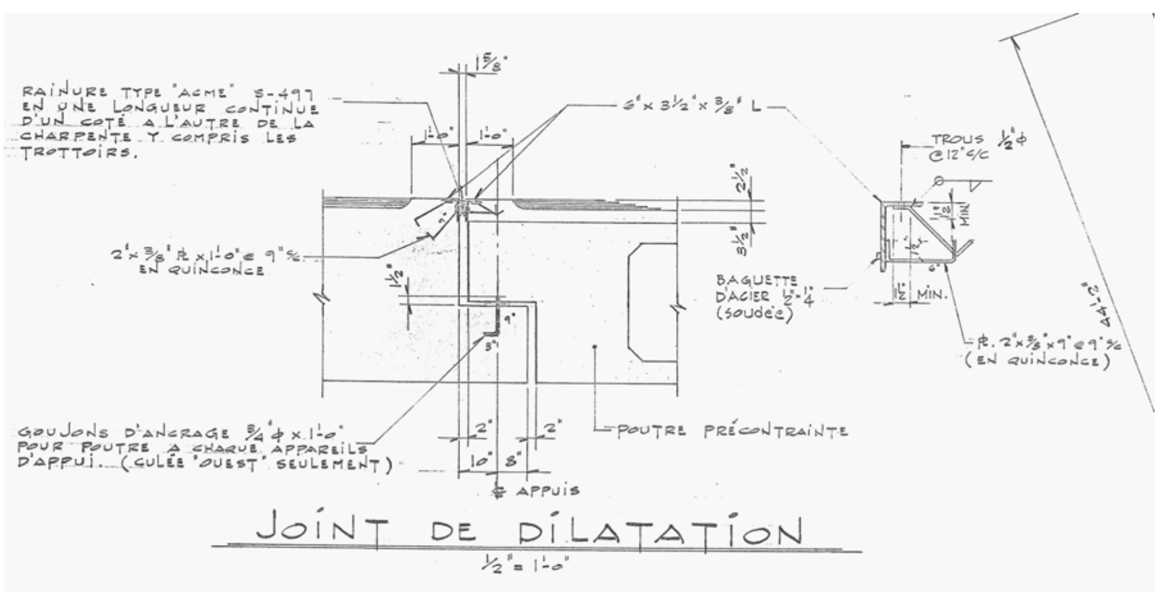

Figure 4.2 – Details of pins and expansion joint reinforcement after revisions (COM-19, p. 13)

- the details of the U-shaped #8 vertical hanger reinforcement near the beam seat was changed (see Fig. 4.3). The total length of the bar was shortened from 11'-6" to 11'-0". Both drawings indicate that the #8 hangers and diagonal #6 bars are hooked around #7 bars. The separation shown between the hooks and # 14 bars is a drafting convention which allows the actual shape of the diagonal bars and the hangers to be shown. The hooks are shown below the #14 bars but above the #7 bars in the regions of the hooks. This should be viewed as a schematic representation. The intent of this drafting detail is that the hooks of the #8 hanger reinforcement and the hooks of the #6 diagonal bars are to be all on the same plane and hooked around the #7 transverse bars, which in turn are immediately below the #14 bars. A cross section of this important detail would have been useful to clarify the intent of the bar details.

- No "bordereau d'armature" (bar list) was recovered but with standard bar bends and an free end extension on the hooks of 18 in., the total height (outside-to-outside) of the U-shaped hanger bars was calculated to be 40 in. The bend diameters, hook lengths and overall height were confirmed by measurements taken at the evidence site and the overall length of the hanger bar was 11'-0".

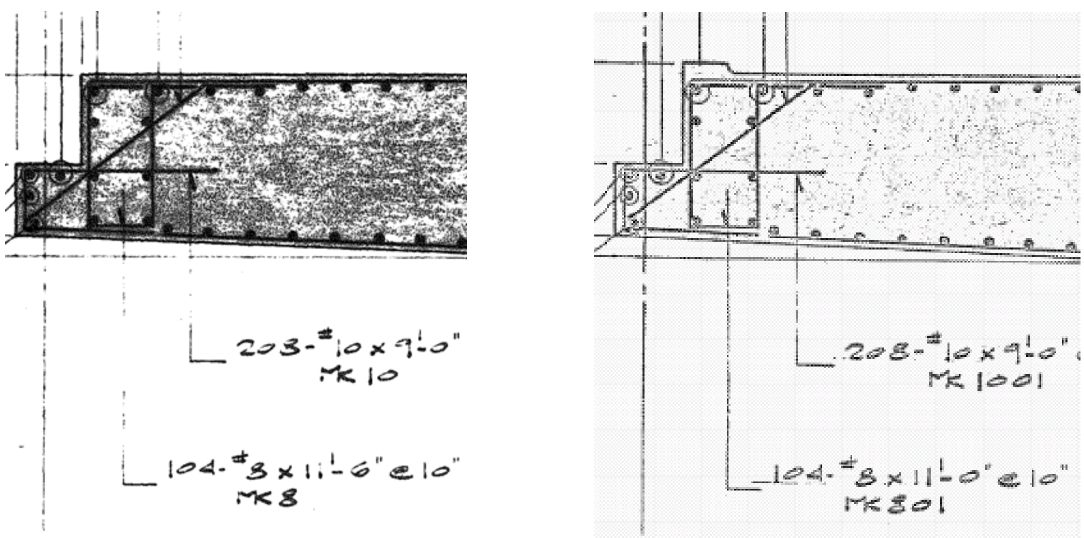

Figure 4.3 – Details of #8 hanger reinforcement on the East side cantilever.
Left side shows details on drawings marked "tel que construit" (COM-19, p. 9)
and right side shows details on revised drawing (COM-19, p. 15)

The following other observations are made relative to the design drawings:

- The concrete cover was not indicated on the structural drawings, where this information is most useful for the personnel at the construction site.
- The yield strength of the reinforcement was not indicated on the drawings.
- The details of the beam seats did not allow for accessibility for inspection and possible maintenance. CSA Standard S6-2006 "Design of Highway Bridges" in Clause 1.8.3.1 states that "The type of access needed for inspection and maintenance shall be considered in the design of all structures. Types of structures that have inaccessible areas where undetected dangerous deterioration can occur shall be avoided".

4.2 Drainage

The following remarks are made with respect to the drainage of the bridge deck:

- The slope of the cantilever for drainage in the transverse direction of Concorde varied from 0.8% to 1.2%. CSA Standard S6-2006 "Design of Highway Bridges" in Clause 1.8.2.2.1 requires "a minimum 2% transverse crossfall". There is no requirement on drainage in the S6-1966 code.

- The longitudinal slope of the bridge is 0.4 %. In current practice, for example as stated in MTQ's "Manuel de conception des structures, volume 1", 2004 edition (CEVC016373, page 016422), this would be considered insufficient, as the necessary minimum is stated as 0.5%.

Conclusions: The revised structural drawings had some missing information. In particular, a cross section of the critical region of the vertical #8 U-shaped hanger bars should have been included to clarify the details intended by the designer. The drawings stamped "tel que construit" were not the final drawings and revisions were made to the drawings and the actual construction. The Concorde bridge deck has insufficient slopes for drainage, both in the transverse and longitudinal directions. The regions at the beam seats, which were directly below the expansion joint, were inaccessible for proper inspection and maintenance.

4.3 Design Check on Bearings

4.3.1 Bearing Stresses

The maximum unfactored reaction (service load) on a typical neoprene pad is estimated as 98.7 kips, including dead loads, live load and impact. Therefore the stress on the neoprene pad is:

$$= \frac{98.7 \times 1000}{6 \times 14} = 1175\,\text{psi}$$

The allowable stress in bearing on the concrete (Clause 8.3.3 (d)) being $= 0.33 f_c' = 0.33 \times 4000 = 1333\,\text{psi}$, is therefore not exceeded.

The allowable bearing stress on unconfined elastomeric bearing pads (Clause 11.2.6) is 800 psi for dead load, live load and impact. This stress is exceeded.

Under dead load the stress on the elastomeric pad is $65.8/(6 \times 14) = 783\,\text{psi}$ which exceeds the allowable stress for dead load only of 500 psi.

Conclusions: The bearing stresses are exceeded on the elastomeric bearing pads, but are within acceptable limits for concrete bearing.

4.3.2 Minimum Pad Thickness Required for Temperature Movement

Clause 11.1 requires that "for structures which are not designed to withstand stresses arising from expansion and contraction shall be designed so as to allow for a total thermal movement at a rate of 1-1/4 inches in a 100 feet. For the 90 ft box girder span with one end fixed, this would require a total movement at the mobile end of $(90/100) \times 1.25 = 1.125\,\text{in.}$ between extreme temperature conditions. The S6-1966 code does not mention the reference temperature, nor the maximum or minimum temperatures to be considered. From current requirements, we would consider the reference temperature at 15°C and approximately 2/3 of the total movement would be contraction from this reference position, and1/3 expansion.

In Clause 11.3.2(b), it is required that the total movement caused by temperature change in either direction from the central position shall not exceed one-half the thickness of the elastomeric material. The total thickness of the neoprene is one inch and hence the movement from the central position must not exceed 0.5 in. The pad is not thick enough to accommodate the total thermal movement of 1.125 in. The pad must accommodate movement arising from thermal effects as well as shrinkage and creep of the precast pretensioned girders.

Conclusions: The bearing pads are not thick enough to accommodate the anticipated movements.

4.4 Design Check of Hanger Reinforcement

The hanger reinforcement is needed to lift the force from the reaction on the pad to the top of the cantilever. This region near the bearing pad and the expansion joint is a region near a concentrated load and an abrupt change in the geometry of the cross section. These regions are recognized by current codes as “disturbed regions” due to the complex flow of stresses in these regions.

The area of vertical hanger reinforcement required, assuming intermediate grade steel with an allowable stress of 20,000 psi (Clause 8.3.3) is:

$$A_{s,required} = \frac{98.7}{20} = 4.94\,\text{in}^2$$

The area of steel provided in a 4 foot width (double legged #8 hanger bars @ 10 in. spacing):

$$A_{s,provided} = \frac{48}{10} \times 2 \times 0.79 = 7.58\,\text{in}^2$$

It is noted that the inclined reinforcement (#6 bars @10 in. spacing) would also contribute as “hanger reinforcement”.

The amount of vertical hanger reinforcement specified on the drawings is sufficient.

It is noted that the anchorage details at the top of the hanger reinforcement are inadequate because the hooks of the #8 hanger bars are not anchored around the longitudinal #14 bars as would be required by current codes. However, the CSA S6-1966 code gave no provisions for the design of “disturbed regions”, with complex flow of stresses.

Conclusions: The anchorage provided by the hooks for the #8 U-shaped hanger reinforcement would not satisfy the requirements for the design of “disturbed regions” given in current codes. The detail shown on the structural drawings creates a weak plane at the top of the #8 hooks of the hangers, where the concrete is relied upon to transfer the forces. It is noted that no provisions were provided in the S6-1966 code for the design of “disturbed regions”.

4.5 Design check of Reinforcement in Nib of Beam Seat

The nib resists the forces from the reaction in the neoprene bearing pad by direct strut action between the neoprene bearing surface and the bottom of the hanger reinforcement.

Assuming a 45° spreading from the 14 in. wide neoprene bearing pad to the centre of the #10 U-bars (2 in. of cover) gives an effective width of $14 + 2(2 + 1.27/2) = 19.27$ in. Thus the number of #10 bars, spaced at 5 in., resisting the moment is 4 bars. Moment to be resisted at face $= 98.7\,\text{kips} \times 10\,\text{in.} = 987\,\text{kip in.}$

To resist this moment the maximum concrete stress is 450 psi and the tensile stress in the top reinforcing bars is 11,900 psi. These stresses are within the allowable stress limits of $0.4 f_c' = 1600\,\text{psi}$ for concrete in compression and 20,000 psi for the reinforcing steel in tension.

Conclusions: The reinforcement in the nib satisfies the code requirements for flexure. The nib does not have any crack control reinforcement as required for the design of “disturbed regions” in later codes.

4.6 Design check of Cantilever for Shear and Moment

4.6.1 Shear

From Section 1.4.3, the total shear due to dead load and H20-S16 truck loading plus impact is 133.8 kips at the face of the wall support.

The effective depth, $\text{d} = 50.875 - 2 - 1.693/2 = 48.0\,\text{in.}$

Shear stress (CSA S6-1966), $v = \dfrac{V}{bd} = \dfrac{133.8 \times 1000}{48 \times 48} = 58\,\text{psi}$

Maximum shear stress for beams without web reinforcement

$$= 1.1\sqrt{f_c'} = 1.1\sqrt{4000} = 69.6\,\text{psi}$$

Conclusions: The calculated shear stress indicates that web reinforcement is not required. This issue is discussed furthering Section 5.

4.6.2 Moment

From Section 1.4.3, the total moment due to dead load and H20-S16 truck loading plus impact is 1512 ft kips at the face of the wall support.

To resist this moment the maximum concrete compressive stress is 1153 psi and the tensile stress in the top reinforcing bars is 23,200 psi. The concrete stress is within the allowable stress limits of $0.4 f_c' = 1600\,\text{psi}$ for concrete in compression. The predicted steel stress is about 16% greater than the allowable stress of 20,000 psi for the reinforcing steel in tension and would be acceptable with some spreading of the live load moments.

Conclusion: The flexural reinforcement is sufficient.

5. 3D Analysis of the de la Concorde Bridge

The purpose of this section is to present the assumptions and results from a three-dimensional analysis of de la Concorde bridge.

The analysis was carried out using the general purpose structural analysis program SAP.

5.1 Description of the model

The analytical model is illustrated in Figure 5.1. The simply supported central span has two independent deck sections due to a longitudinal joint along the centre of the bridge. Each of these separate decks is modelled as a monolithic multi-cell box girder using plate elements. The thickness of the concrete topping placed over the prefabricated girders is included in the definition of the top flange of the deck elements. The sidewalks and central median are included as non structural elements with very low stiffness but their self weight is fully included. The nominal thickness of the asphalt paving is considered by an applied uniform load.

The bearing pads are modelled as vertical bar elements with a very small horizontal stiffness and representative vertical stiffness. The West abutment is represented by a cantilever slab, clamped at the top of the inclined wall. The East abutment is represented in detail, including the variable thickness of the deck slab, the inclined front wall, the vertical back wall and the four longitudinal walls of trapezoidal shape. The displacements of the base of the front wall are fixed and the tie-down anchors are simulated by blocking the vertical displacement at the far end of each longitudinal wall.

The following mechanical properties were selected:

Material self weight properties according to CSA S6-2006
Concrete assumed uncracked, E_c= 25 GPa
Neoprene pads (6” x 14” x 1” thickness, Duro 60), E_n= 69 MPa, in accordance with ASSHTO requirements

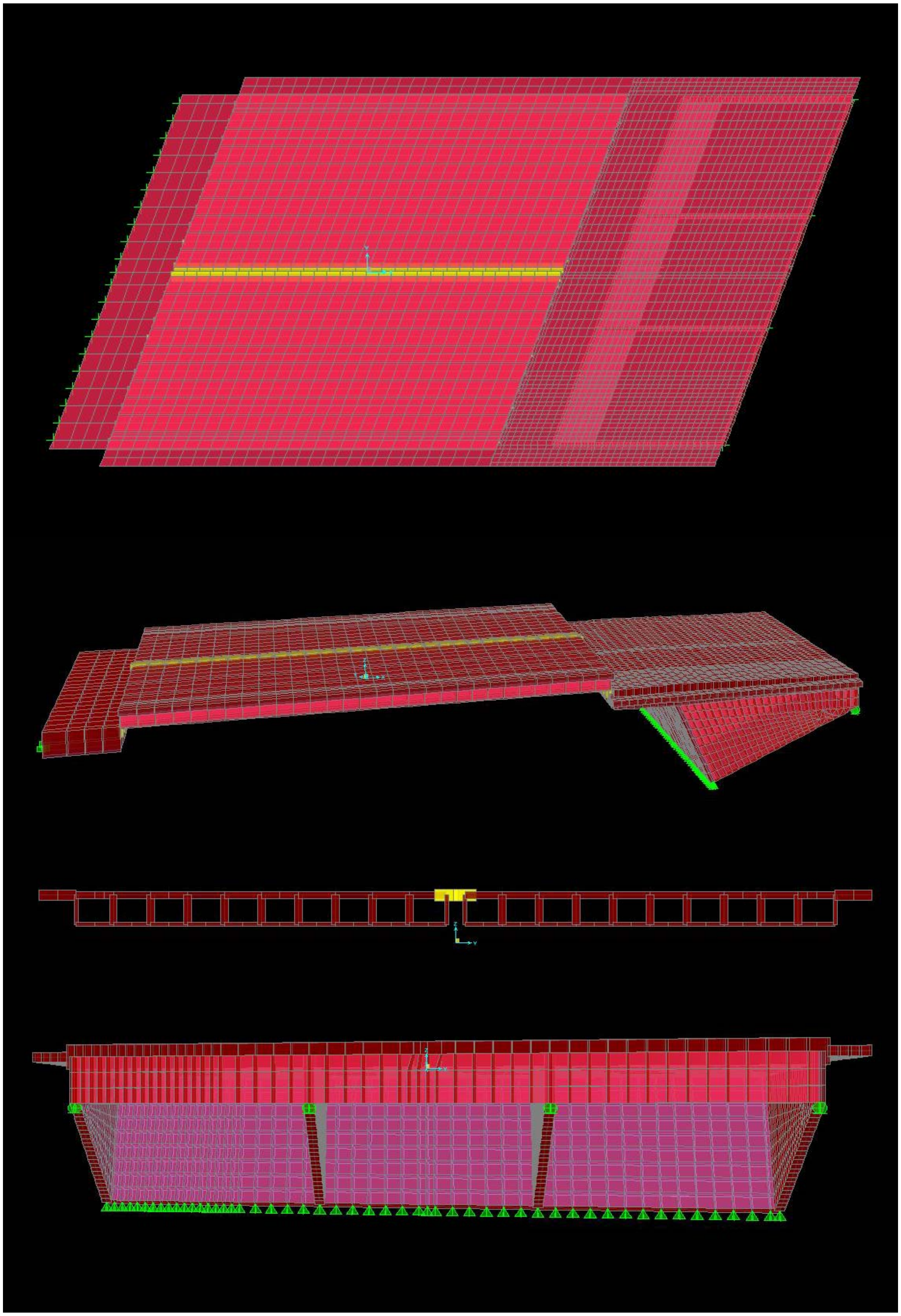

Figure 5.1 – Illustrations of the 3-D analysis model: plan view, general view, cross section of the central span and view of the East abutment seen from the back side

5.2 Modelling of loads

Since the girders are prefabricated and placed on the bearing pads before the transverse prestressing of the girders and placement of the 3 ½" concrete topping, the weights of the girders and topping are applied as point loads uniformly distributed over all 40 bearing pads. As for the live loads, the weights of the sidewalks, median and asphalt paving are applied to the deck acting as monolithic, so as to properly distribute these load effects.

The design truck loads corresponding to the H20-S16 standard truck loading prescribed by the S6-1966 Code as well as those corresponding to the CL625 truck of S6-2006 are considered, acting independently in the outer lane and middle lanes of the eastbound side or South side of the overpass, where the effect of the skew of the bridge is most severe (the same situation also exists at the North-West abutment). It is noted that the total load of the H20-S16 and CL625 trucks are 320 kN and 625 kN, respectively.

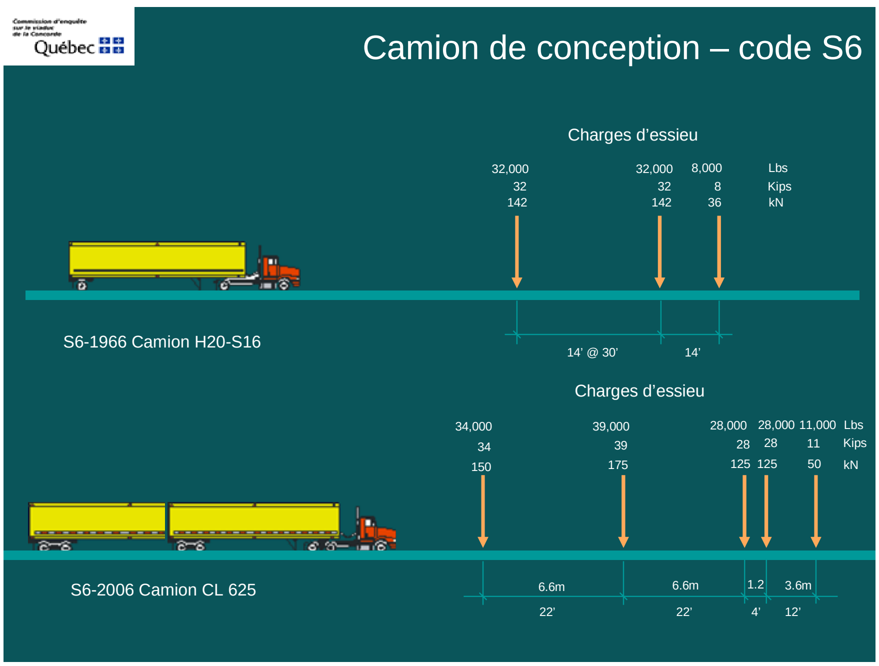

Figure 5.2 – Comparison of H20-S16 and CL625 standard trucks

The load cases included in the reported analysis results are given in Table 5.1.

Table 5.1 – Description of loading cases

Load case identification	**Description**
D	Dead load (all unfactored dead loads included)
DSWMED	Dead load for sidewalks and median only
L625CE3	Live load from CL625: truck on centre span, centre lane, front axle at expansion joint
L625CE4	Live load from CL625: truck on centre span, centre lane, second axle at expansion joint
L625DR3	Live load from CL625: truck on centre span, outer lane
L625DR4	Live load from CL625: truck on centre span, outer lane, front axle at expansion joint
LHSCE	Live load from H20-S16: truck on centre span, centre lane, front axle at expansion joint
LHSDR	Live load from H20-S16: truck on centre span, outer lane, front axle at expansion joint
LSIDEW	Live load on sidewalks (3.2 kPa = 67 lbs/ft^2)
Note: all loads are unfactored and live loads do not include dynamic load amplification (DLA)	

5.3 Force distribution on the bearing pads

Figure 5.3 illustrates the location of the bearing pads of the East abutment. Bearing pad 21 is the outer pad on the South side, pads 30 and 31 are the innermost pads, while pad 40 is the outer pad on the North side. The corresponding bearing pad reactions are presented in Table 5.2.

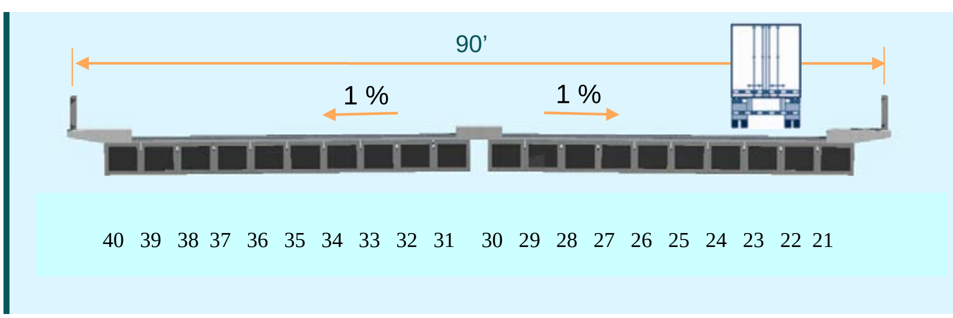

Figure 5.3 – Identification of bearing pads - Concorde

Table 5.2 – Summary of bearing pad reactions - SAP Model VC11 - Concorde

PAD no.	21	22	23	24	25	26	27	28	29	30	CUMUL
Output Case	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)
D	-398	-353	-321	-306	-294	-284	-277	-269	-256	-239	-2997
DSWMED	-62	-41	-27	-21	-16	-14	-12	-12	-11	-10	-227
L625CE3	-32	-29	-27	-28	-27	-23	-19	-14	-9	-5	-211
L625CE4	-27	-26	-26	-30	-31	-28	-22	-15	-10	-6	-220
L625DR3	-160	-118	-81	-56	-36	-21	-10	2	19	42	-420
L625DR4	-151	-120	-87	-62	-40	-24	-12	-1	14	35	-448
LHSCE	-32	-31	-31	-35	-35	-30	-24	-18	-13	-8	-259
LHSDR	-90	-69	-48	-34	-23	-14	-7	-1	8	20	-259
LSIDEW	-54	-35	-20	-13	-7	-3	1	5	11	20	-94

PAD no.	31	32	33	34	35	36	37	38	39	40	CUMUL
Output Case	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)
D	-359	-323	-298	-286	-279	-277	-277	-282	-297	-319	-2997
DSWMED	-34	-23	-16	-13	-12	-14	-17	-22	-32	-45	-227
L625CE3	2	0	0	-1	-1	-1	0	0	0	0	0
L625CE4	2	0	0	-1	-1	-1	0	0	0	0	0
L625DR3	2	0	0	0	0	0	0	0	0	0	0
L625DR4	2	0	0	-1	-1	-1	0	0	0	0	0
LHSCE	2	0	0	-1	-1	-1	0	0	0	1	0
LHSDR	1	0	0	0	0	0	0	0	0	0	0
LSIDEW	17	7	1	-3	-6	-9	-13	-18	-28	-41	-94

5.4 Comparison of dead load pad reactions for 3D analysis and simplified analysis

Table 5.3 compares the dead load reactions from the 3D analysis and from a simplified analysis using hand calculations on the 10 pads on the south side of the bridge. In the simplified analysis it was assumed that 75% of the sidewalk and median dead loads were distributed into the outermost girders and the remaining 25% were distributed into the first interior girders.

Table 5.3 – Comparison of dead load reactions in kN - Concorde

PAD no.	21	22	23	24	25	26	27	28	29	30	Sum
3D Analysis	398	353	321	306	294	284	277	269	256	239	2997
Simplified	419	338	274	274	274	274	274	274	299	321	3019

5.5 Comparison of pad reactions due to truck loading for 3D analysis and simplified analysis

The simplified method of the CSA S6-1966 Code gives uniform pad reactions of 25.3 kips (113 kN) for the H20-S16 truck loading without the impact factor. The 3D analysis (see Table 5.2) gives considerably different distributions of pad reactions for the H20-S16 loading than the simplified analysis. For the truck placed on the outside lane and in the centre lane of the eastbound lanes, the maximum pad reactions were 90 kN and 32 kN, respectively. When combining simultaneous live loads in the outer and centre lanes, the total pad reaction is 90 + 32 = 122 kN, about 8% more than obtained from the simplified analysis.

The simplified method of the CSA S6-2006 Code gives uniform pad reactions of 244 kN for the truck loading with the Dynamic Load Amplification Factor, which corresponds to 195 kN without this factor. The 3D analysis (see Table 5.2) gives different distributions of pad reactions for the CL625 loading than the simplified analysis. For the truck placed on the outside lane and in the centre lane of the eastbound lanes, the maximum pad reactions were 160 kN and 32 kN, respectively. When combining simultaneous live loads in the outer and centre lanes, the total pad reaction is 160 + 32 = 192 kN, essentially equal to that obtained from the simplified analysis.

For all traffic load cases, due to the H20-SS16 or CL625 design truck, the 3D analysis gives greater transverse distribution of the truck loading. This was observed for every position of the truck considered. In addition, the 3D analysis accounts for the stiffness of the superstructure, cantilever and abutment.

5.6 Summary of stress resultants from 3D analysis

The stress resultant results are provided at a series of elements in the cantilever slab located at the face of the inclined wall, and approximately "d" away from the face. These elements are shown in Figure 5.4 and the stress resultants are presented in Table 5.4.

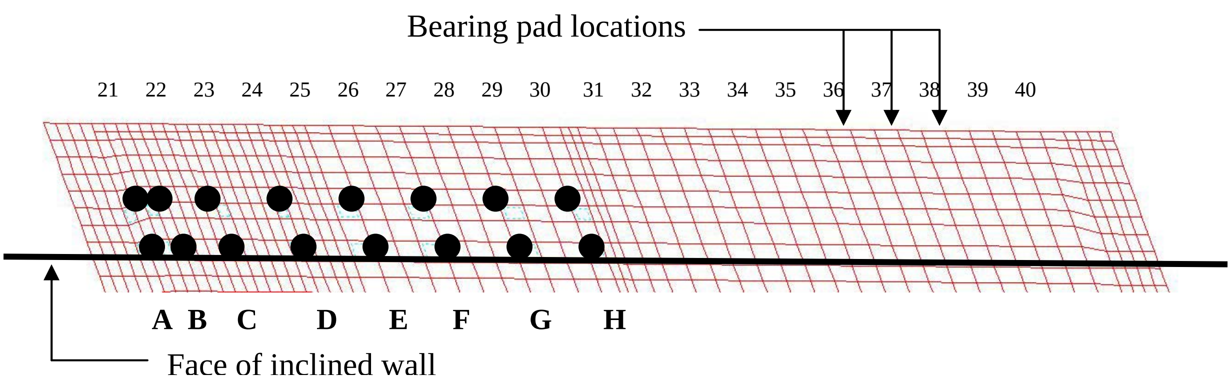

Figure 5.4 – Location of stress resultants in cantilever slab - Concorde

Figure 5.5 illustrates the distribution of the principal bending moments (per unit width of the slab) measured along the longitudinal axis of the bridge (parallel to the traffic and parallel to the direction of the main reinforcing steel in the cantilever). It should be noted that the critical location for bending, which should govern the design of the top steel, is at the face of the inclined wall.

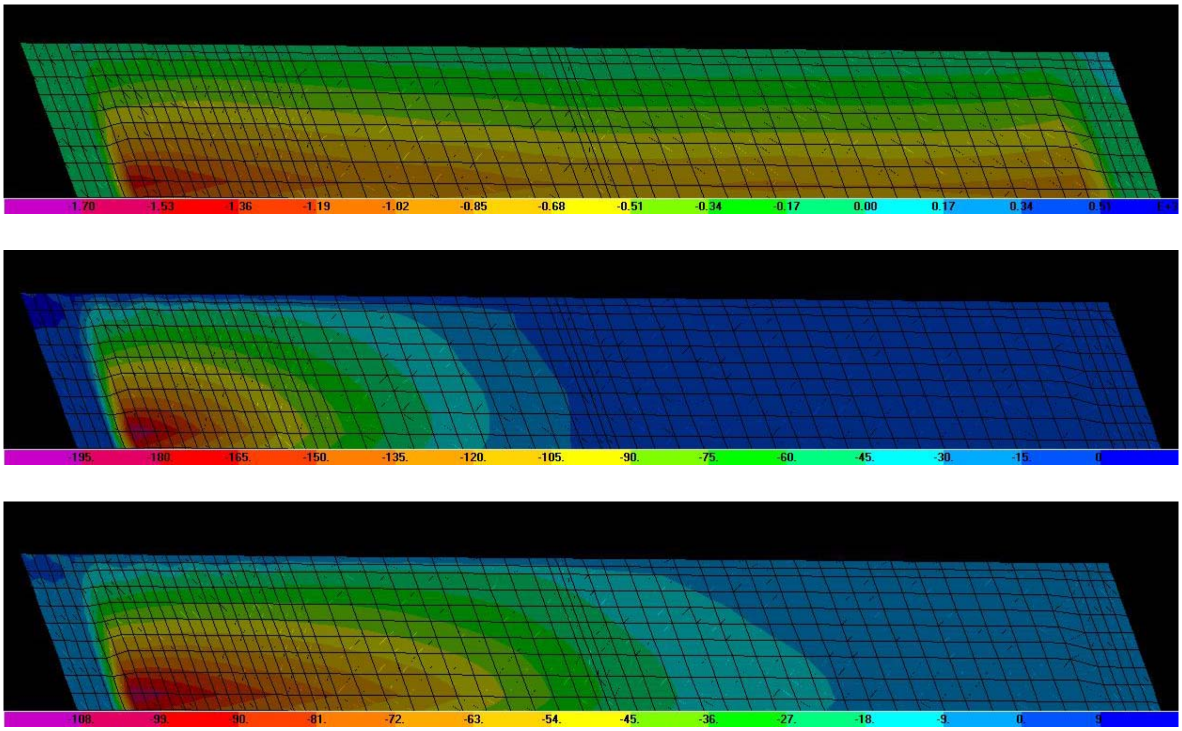

Figure 5.5 – Distribution of principal bending moment (unit (kN-m/m)
(top: dead load; middle: H20-S16 in outer lane, South side;
lower: H20-S16 in centre lane, South side)

Figure 5.6 illustrates the distribution of the maximum shearing force (per unit width of the slab). It should be noted that the shearing force is critical a distance “d” away from the face of the inclined wall, not at the face of the wall, because of the presence of high compressive stresses at the face. It is noted that “d” is the effective depth of the slab, measured from the centre of gravity of the #14 top steel to the bottom face of the slab.

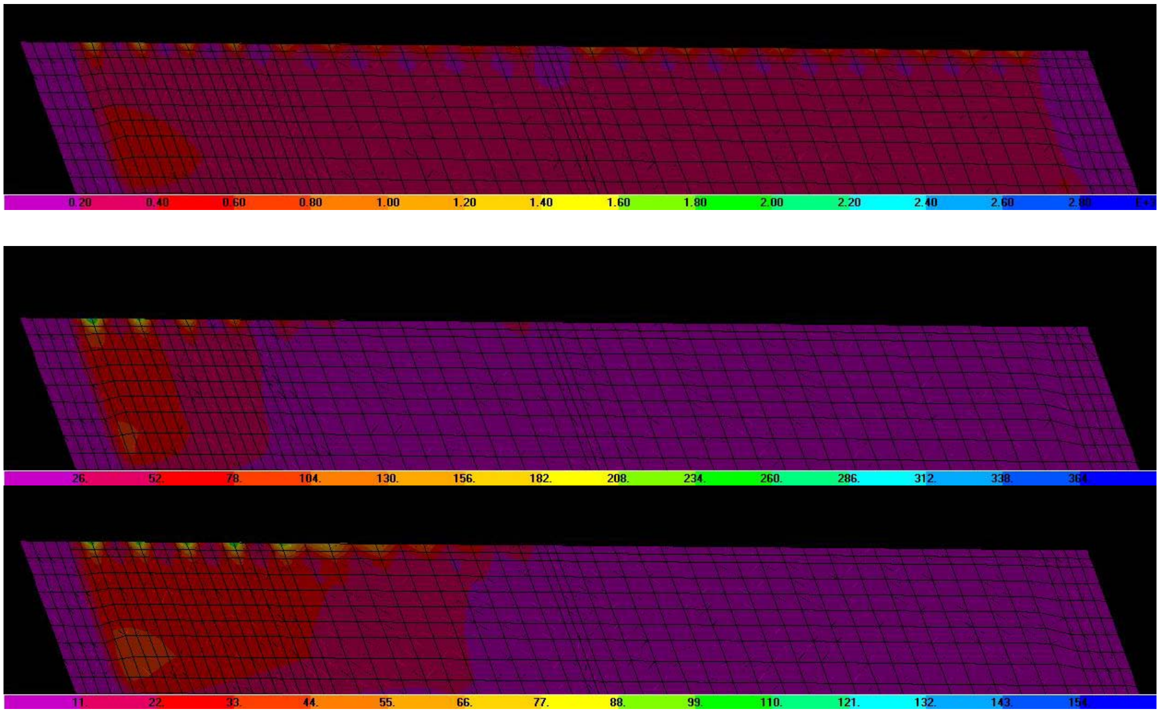

Figure 5.6 – Distribution of shear force (unit (kN/m)
(top: dead load; middle: H20-S16 in outer lane, South side;
lower: H20-S16 in centre lane, South side)

Table 5.4 – Summary of stress resultants in selected elements of the cantilever slab

Element	Distance from center line of bridge	OutputCase	Stress resultants at face of inclined wall M11 max	Stress resultants at face of inclined wall Vmax		Stress resultants at "d" from face of wall M11 max	Stress resultants at "d" from face of wall Vmax
	feet		KN-m/m	KN/m		KN-m/m	KN/m
H	-1.5 to -3	D	879	309		582	261
		DSWMED	205	108		107	73
		L625CE3	25	3		23	3
		L625CE4	28	4		26	4
		L625DR3	23	10		24	11
		L625DR4	30	9		30	9
		LHSCE	32	5		29	5
		LHSDR	17	5		17	5
		LSIDEW	8	9		6	8
G	-7 to -9	D	931	295		631	272
		DSWMED	207	102		113	76
		L625CE3	36	7		32	7
		L625CE4	41	9		36	9
		L625DR3	46	12		48	11
		L625DR4	54	10		53	8
		LHSCE	46	10		40	10
		LHSDR	31	6		30	5
		LSIDEW	5	9		8	8
F	-15 to -17	D	1003	310		685	277
		DSWMED	234	107		135	75
		L625CE3	51	12		42	13
		L625CE4	58	17		44	17
		L625DR3	90	12		87	9
		L625DR4	98	13		93	12
		LHSCE	64	16		52	17
		LHSDR	56	7		52	7
		LSIDEW	19	7		22	4
E	-21 to -23	D	1052	311		738	292
		DSWMED	267	111		156	80
		L625CE3	61	17		49	18
		L625CE4	66	22		49	21
		L625DR3	132	27		124	27
		L625DR4	142	30		130	31
		LHSCE	74	21		59	22
		LHSDR	80	17		74	17
		LSIDEW	34	7		34	6

Table 5.4 – Summary of stress resultants in selected elements of the cantilever slab (continued)

Element	Distance from center line of bridge	OutputCase	Stress resultants at face of inclined wall			Stress resultants at "d" from face of wall	
			M11 max	Vmax		M11 max	Vmax
	feet		KN-m/m	KN/m			feet
D	-27 to -28	D	1132	346		795	324
		DSWMED	289	117		171	87
		L625CE3	70	21		55	21
		L625CE4	73	25		54	24
		L625DR3	184	49		163	50
		L625DR4	194	53		168	54
		LHSCE	83	25		63	26
		LHSDR	109	29		96	30
		LSIDEW	51	14		48	14
C	-32 to -33	D	1254	413		856	383
		DSWMED	309	128		181	102
		L625CE3	81	27		59	26
		L625CE4	81	27		57	26
		L625DR3	249	84		202	83
		L625DR4	258	90		203	87
		LHSCE	93	31		67	30
		LHSDR	146	49		118	49
		LSIDEW	77	28		64	26
B	-38 to -39	D	1452	674		837	444
		DSWMED	344	214		170	121
		L625CE3	96	43		59	30
		L625CE4	92	39		55	28
		L625DR3	340	178		217	121
		L625DR4	345	182		216	121
		LHSCE	108	46		65	33
		LHSDR	197	102		126	70
		LSIDEW	116	70		72	43
A	-40 to -41	D	1442	703		880	451
		DSWMED	331	235		177	125
		L625CE3	96	44		62	31
		L625CE4	92	40		58	28
		L625DR3	338	183		231	124
		L625DR4	342	187		229	124
		LHSCE	107	47		68	33
		LHSDR	196	105		133	71
		LSIDEW	114	74		77	45

5.7 Concluding remarks on 3D analysis

The analytical results illustrate the importance of the skew of the bridge in the distribution of internal stresses in the cantilever slab. In addition the influence of the cantilevered sidewalk increases the stress resultants near the south side. The stiff abutment walls also results in more concentration of the stress resultants in the regions of the walls.

6. Comparisons of Factored Resistances and Factored Loads - de la Concorde Bridge – S6-2006 code checks

The factored loads obtained from the 3D finite element analysis are compared with the factored resistances using the load factors (see Section 2) and the general shear design method from CSA S6 2006. Because the results from the 3D analysis are more representative of the actual behaviour of the bridge, the results from this analysis were used in assessing the factored loads on the cantilever.

Table 6.1 summarizes the factored moments and shears in the more highly stress region of the cantilever near the SE corner (near pads 21, 22 and 23). In calculating the factored shear resistance, V_r, and the flexural resistance, M_r, the procedures described in CSA S6-2006 were used. For the calculations of V_r, the General Method for shear design of Section 8.9.3.7 was used. This method accounts for the “size effect” in shear, for the interaction between shear and moment and the influence of the amount of flexural reinforcement. In addition, the vertical component of the inclined compression parallel to the bottom face of the cantilever was considered. For these calculations, the maximum moments and shears per metre width were factored with the appropriate load factors to get the factored load effects. The factored moment and shear resistances were also determined for a one metre width. The maximum factored moment at the face of the inclined wall and the factored shear and co-existing factored moment at a distance of d from the face of the wall were used.

Table 6.1 – Comparison of factored loads and factored resistances – de la Concorde

Element/ Location	M_f kNm/m	M_r kNm/m	M_r/M_f	V_f kN/m	V_r kN/m	V_r/V_f	Comments
A at face							
CL625	2696	2760	1.02				OK
H20-S16	2438	2760	1.13				OK
B at face							
CL625	2717	2760	1.02				OK
H20-S16	2455	2760	1.12				OK

Element/ Location	M_f kNm/m	M_r kNm/m	M_r/M_f	V_f kN/m	V_r kN/m	V_r/V_f	Comments
C at face							
CL625	2225	2760	1.24				OK
H20-S16	2035	2760	1.36				OK
A at d							
CL625	1705			891	576	0.65	NG
H20-S16	1531			794	601	0.76	NG
B at d							
CL625	1615			876	584	0.67	NG
H20-S16	1354			782	620	0.79	NG
C at d							
CL625	1598			703	606	0.86	NG
H20-S16	1449			637	628	0.99	Say OK

As can be seen from Table 6.1, the factored flexural resistance is sufficient. However, the factored shear resistance is insufficient for both the CL635 loading cases and H20-S16 truck loading in the 10 foot outer portion of the cantilever slab (South side of East cantilever, or North side of West cantilever).

Because the factored shear exceeds the factored shear resistance, stirrups must be provided and at least minimum transverse reinforcement must be provided. With the cracking strength of concrete defined as :

$$f_{cr} = 0.4\sqrt{f_c^{'}} = 0.4\sqrt{27.58} = 2.10\,\text{MPa},$$

from S6-2006, clause 8.9.1.3, the minimum reinforcement A_v shall not be less than:

$$A_v = \frac{0.15 f_{cr} b_v s}{f_y}.$$

If Grade 400 stirrups are used with stirrup legs on a grid of 600 mm by 600 mm, then A_v required is:

$$A_v = \frac{0.15 \times 0.4 \times \sqrt{27.58} \times 600 \times 600}{400} = 284\,\text{mm}^2$$

One 20M bar has an area of 300 mm^2 and hence 20M stirrup legs at on a grid spacing of 600 x 600 mm would satisfy the minimum reinforcement requirements.

The location at d from the wall support with the highest value of V_f/V_r requires a factored shear resistance of 891 kN/m. The factored shear resistance for a unit width of slab (1000 mm) that contains minimum transverse reinforcement (20M on a grid spacing of 600 x 600 mm) can be determined as follows:

Location A has the highest shear and for the factored shear and moment on the critical section (for the case of CL625 loading) the following resistances can be determined:

$V_c = 536\,\text{kN}$

$V_p = 40\,\text{kN}$

The additional resistance provided by the stirrups is:

$$V_s = \frac{\phi_s f_y A_v d_v \cot\theta}{s} = \frac{0.9 \times 400 \times 500 \times 1062 \times \cot 43.8}{600} = 332\,\text{kN}$$

$$V_r = V_c + V_p + V_s = 536 + 40 + 332 = 908\,\text{kN}\ .$$

Therefore the minimum stirrups are sufficient to provide the required strength in shear.

An assessment was also made in this critical region of the cantilever, but near the beam seat. Minimum stirrups are required in this region as well.

In conclusion, an assessment of the cantilever indicates that in accordance with the design provisions of CSA S6-2006, minimum shear reinforcement is required in the cantilever. This conclusion holds true using the H20-S16 as well as the CL625 live load.

7. Code Requirements for Shear

This section addresses the code requirements for shear with respect to the following aspects and compares these requirements to the design of the de la Concorde overpass:

- The need for shear (transverse) reinforcement
- Minimum shear reinforcement
- The difference between requirements for beams and for slabs
- Influence of depth of the element

Key aspects of the provisions of North American codes in force at the time of the design are presented. These include:

- Canadian Standards Association CSA Standard S6-1966 “Design of Highway Bridges”. This code was used for the design of the bridge
- American Association of State Highway Officials (AASHO) Standard Specifications for Highway Bridges, 1965.
- American Concrete Institute (ACI) “Building Code Requirements for Reinforced Concrete (ACI 318-63), 1963.

The current Canadian bridge design codes are also reviewed:

- Canadian Standards Association CSA Standard S6-2006 “Design of Highway Bridges”. This is the current Canadian code for the design of bridges.

7.1 CSA S6-1966

The specified concrete compressive strength f_c' for this bridge was 4000 psi. From Clause 8.3.2(b), The maximum shear stress in “beams without web reinforcement” is $1.1\sqrt{f_c'} = 70\,\text{psi}$. In assessing the design it was assumed that the cantilever can be designed as a series of 4 ft wide equivalent beams and assuming that in the critical region near the south east face a suitable cantilever span is 13 ft (due to the skew supports for the girders). The equivalent beam analysis of Section 4 gives a shear stress of 58 psi due to unfactored loads.

Using the results of the more refined 3D analysis at outer element H of Figure 5.4, at a distance of d from the support face, we find that the loads would be as follows (see Table 5.3): the dead load is 451 kN/m, the H20-S16 truck load in the outer lane is 71 kN/m x 1.3 impact factor and the truck load in the centre lane is 33 kN/m x 1.3 impact factor. The total shear is:

$$V = 451 + 1.3 \times (71 + 33) = 586\,\text{kN/m} = 40.1\,\text{kips per ft width}$$

The shear stress is:

$$v = \frac{40.1 \times 1000}{12 \times 48} = 70 \text{ psi}$$

just at the allowable limit. If the live load effect on the sidewalk is added, similar calculations would yield shear stresses of 75, 74, 61 and 49 psi at elements A, B, C and D, respectively. Hence, using the more refined analysis results, we find that the concentrations of shear in these critical regions produce shear stresses just above the limiting shear stress under working loads in accordance with S6-1966.

Conclusion: The calculations, using analysis procedures used at the time of the design, indicate that no transverse reinforcement (stirrups) would be required in the cantilever. However modern 3D analysis techniques, some shear reinforcement would be required.

7.2 AASHO-1965

The maximum shear stress in "beams without web reinforcement" when the longitudinal bars are anchored is $0.03 f_c' = 0.03 \times 4000 = 120 \text{ psi}$ but limited to a maximum stress of 90 psi. The calculated shear stresses under working loads are below 90 psi.

Conclusion: The calculations indicate that no transverse reinforcement (stirrups) would be required for shear in the cantilever, under AASHO-1965 requirements.

7.3 ACI 318-1963

Clause 1201 of the 1963 ACI Code states that "The shear stress, v_c, on an unreinforced web shall not exceed $1.1\sqrt{f_c'}$ at a distance d from the face of the support unless a more detailed analysis is made…". The maximum shear stress permitted is 70 psi which exceeds the maximum calculated shear stress.

Clause 1202 requires that "wherever the value of the shear stress, v, exceeds the shear stress, v_c, permitted for the concrete of an unreinforced web by Sections 1201 (c), (d) or (e), web reinforcement shall be provided to carry the excess.

This code did not have any minimum reinforcement requirements.

Conclusion: The calculations indicate that no transverse reinforcement (stirrups) would be required for shear in the cantilever, under ACI 318-1963 requirements.

7.4 CSA S6-2006

Clause 8.9.4.1 of the CSA S6-2006 Standard requires that for determining the shear resistance of slabs that one must consider “beam action, with a critical section extending in a plane across the entire width and located at a distance, d, from the face of the concentrated load or reaction area …”.

For solid slabs a minimum amount of transverse reinforcement is not required unless the factored shear force, V_f , exceeds the factored shear resistance, V_r , (Clause 8.9.3.3). For a case without transverse reinforcement the factored shear resistance is equal to V_c , the factored shear resistance provided by tensile stresses in the concrete.

The analysis shown in Sections 5 and 6 indicate that minimum reinforcement would be required.

Conclusion: The calculations indicate that transverse reinforcement (stirrups) would be required for shear in the cantilever. These calculations were carried out with both H20-S16 and CL625 truck loading.

8. Design Procedures and Code Requirements for Regions Adjacent to Concentrated Loads, or Abrupt Changes in Cross Section

The region of the beam seat has a complex flow of stresses due to the concentrated load and the abrupt change in cross section. General procedures for the design of regions adjacent to supports, concentrated loads, or abrupt changes in cross section first appeared in codes in 1984, with the strut-and-tie design provisions in Clause 11.4.7 of the 1984 CSA Standard CAN3-A23.3-M84. This design method also was adopted by the U.S. AASHTO LRFD Bridge Design Specifications in 1994 and 2004 and by the 2000 and 2006 Canadian Standards Association CSA Standard S6-2006 “Design of Highway Bridges”.

It is noted that the U-shaped #8 hanger bars play a key role in providing a tension tie to lift the load from the pad reaction to the top of the cantilever section. CSA A23.3-M84 requires that the tension tie be effectively anchored to transfer the required tension.

The Canadian Standards Association CSA Standard S6-1966 “Design of Highway Bridges” did not have design provisions for these regions with complex stress flows. The vertical #8 U-shaped hanger bars near the beam seat had sufficient area to provide the necessary tension tie force, however these bars had inadequate anchorage at the top of this hanger reinforcement. The hooks of the #8 hanger bars are not anchored around the longitudinal #14 bars.

Conclusion: Inadequate anchorage was provided near the top of the #8 U-shaped hanger reinforcement and this created a weak plane in a crucial part of the cantilever.

9. Comparison of de la Concorde and De Blois Bridges

9.1 Geometry

The differences in the geometry are as follows:

- The main spans of the hollow box girders are both 90 ft but the De Blois bridge has an overall smaller width than the Concorde bridge. For Concorde the traffic lane width in each direction is 35'-6'' with a central 5 ft wide median, while De Blois has a traffic lane width of 23'-0'' in each direction and has no central median.
- Concorde has a longitudinal joint along the centre of the bridge while De Blois has no such joint.
- Concorde has a total of 20 hollow box girders, 10 on each side of the longitudinal joint, while De Blois has a total of 13 hollow box girders without any longitudinal joint.
- The Concorde bridge has skewed supports at the beam seats with the line of bearings forming an angle of 69º-30' from the longitudinal axis of the bridge. De Blois has practically no skew, having its line of bearings at an angle of 89º-59'-15'' from the longitudinal axis of the bridge.
- The Concorde bridge has sidewalks that are 7'-0'' wide while the sidewalks of De Blois are 6'-0'' wide. In addition, the thickness of the sidewalk is slightly less for the De Blois bridge.
- The cantilever span for Concorde was 13'-0'' along the longitudinal axis of the bridge, while for De Blois the cantilever span was 12'-0''.
- The slope of the cantilever for drainage in the transverse direction of Concorde varied from 0.8% to 1.2% while De Blois had a slope from the crown of 1.8%. It is noted that the Canadian Standards Association CSA Standard S6-2006 "Design of Highway Bridges" in Clause 1.8.2.2.1 requires "a minimum 2% transverse crossfall".
- The thickness of the cantilever slab for Concorde is 45.5" at the expansion joint (excluding joint lip) and at the wall support is 50.875" on the east side and 52.125" on the west side. The thickness of the cantilever slab for De Blois at the expansion joint (excluding joint lip) varies from 44.72" to 50.24" on the east side and varies from 45.2" to 50.24" on the west side. For De Blois the thickness of the cantilever slab at the wall support varies from 50.72" to 55.76" on the east side and varies from 51.2" to 56.24" on the west side. These dimensions are similar for the two bridges with the De Blois cantilever thickness varying from 98% to 111% of the thickness of the Concorde cantilever.
- The reinforcing details are very similar with the same size and spacing of the bars with only slight difference on the length of the #10 U-bars and the #6 diagonal

bars. The #14 main longitudinal top bars and the bottom #8 longitudinal bars have the same spacings in both bridges.

9.2 Differences in design

The de la Concorde and De Blois bridges have the same span, have similar dimensions of the cantilever and have the same amount of reinforcement near the beam seat (bar sizes and spacings). The two bridges also have the same bar sizes and reinforcement spacings of the main reinforcement near the beam seat (#8 U-shaped hanger bars, #10 U-bar and #6 diagonal bar). The two bridges also have the same amount of longitudinal reinforcement for flexure (#14 top bars and #8 bottom bars) in the cantilever.

9.3 3D Finite Element Analysis of De Blois Bridge

9.3.1 Model of bridge abutments and central span

The analytical model is illustrated in Figure 9.1. The simply supported central span is modelled as a monolithic multi-cell box girder using plate elements. The thickness of the concrete topping placed over the prefabricated girders is included in the definition of the top flange of the deck elements. According to the drawings, the thickness of the topping varies, from 3”at the curb to 8” at the crown: the model uses a uniform thickness of 5½”. The sidewalks are included as non structural elements with very low stiffness but their self weight is fully included. The nominal thickness of the asphalt paving is considered by an applied uniform load.

The bearing pads are modelled as vertical bar elements with a very small horizontal stiffness and representative vertical stiffness. The West abutment is represented by a fixed support. The East abutment is represented in detail, including the variable thickness of the deck slab, the inclined front wall, the vertical back wall and the three (3) longitudinal walls of trapezoidal shape. The displacements of the base of the front wall are fixed and the tie-down anchors are simulated by blocking the vertical displacement at the far end of each longitudinal wall.

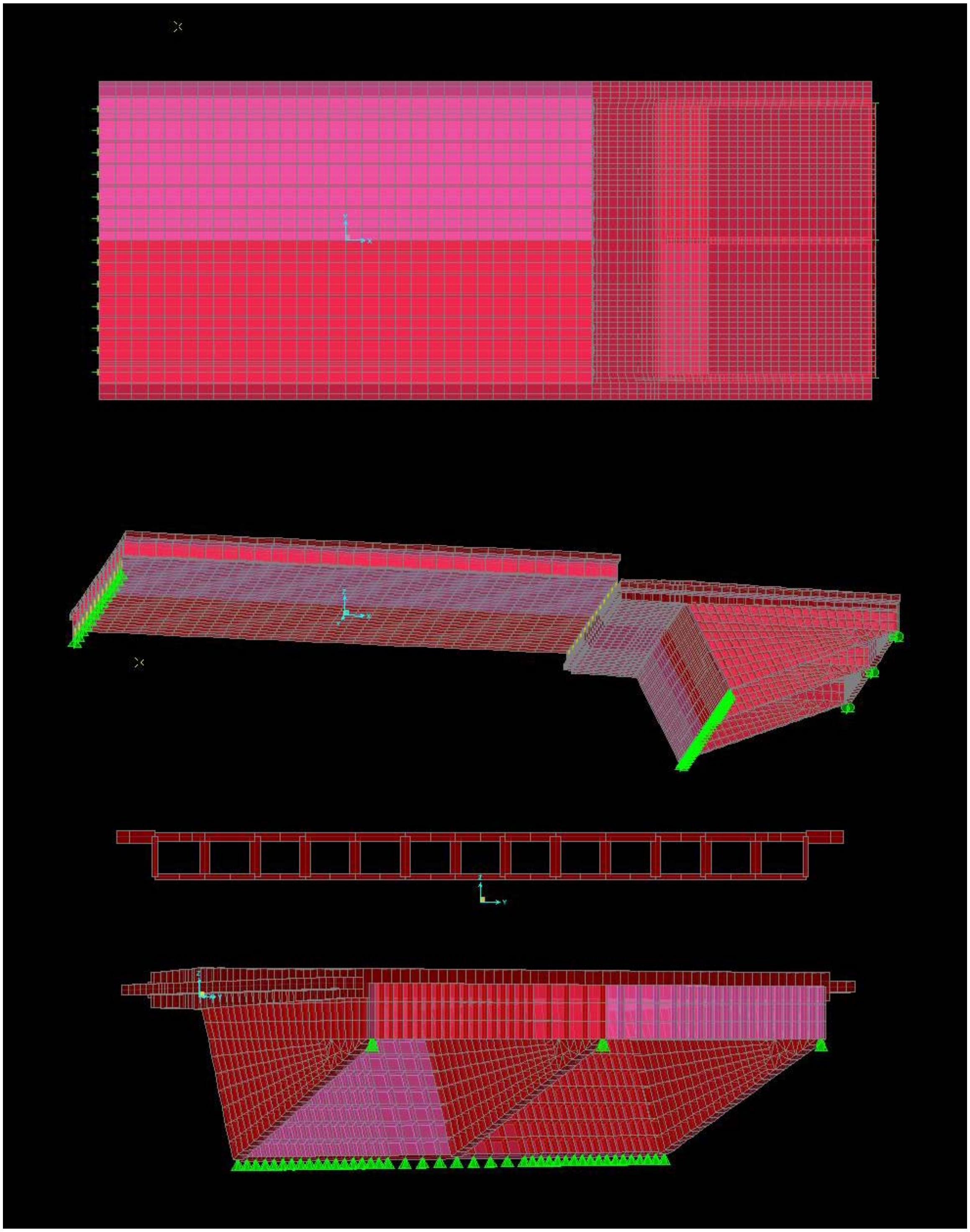

Figure 9.1 – Illustrations of the 3-D analysis model: plan view, general view, cross section of the central span and view of the East abutment seen from the back side

The following mechanical properties were selected:

Material self weight properties according to CSA S6-2006
Concrete assumed un-cracked, E_c= 25 GPa
Neoprene pads (6” x 14” x 1” thickness, Duro 60), E_n= 69 MPa, in accordance with ASSHTO requirements

A similar approach was taken for the modelling of the De Blois Bridge as was taken for the de la Concorde Bridge. A description of the loading cases is given in Table 9.1.

Table 9.1 – Loading cases

Load case identification	**Description**
D	Dead load (all dead loads included)
DSWMED	Dead load for sidewalks and median only
L625CE3	Live load from CL625: truck on centre span, inside lane, front axle at expansion joint
L625CE4	Live load from CL625: truck on centre span, inside lane, second axle at expansion joint
L625DR3	Live load from CL625: truck on centre span, outer lane
L625DR4	Live load from CL625: truck on centre span, outer lane, front axle at expansion joint
LHSCE	Live load from H20-S16: truck on centre span, inside lane, front axle at expansion joint
LHSDR	Live load from H20-S16: truck on centre span, outer lane, front axle at expansion joint
LSIDEW	Live load on sidewalks (3.2 kPa = 67 lbs/ft^2)
Note: all loads are unfactored and live loads do not include dynamic load amplification (DLA)	

9.3.2 Bearing pad reactions

Figure 9.2 illustrates the location of the bearing pads of the East abutment. Bearing pad 1 is the outer pad on the South side, while pad 17 is the outer pad to the North. The corresponding bearing pad reactions are presented in Table 9.2.

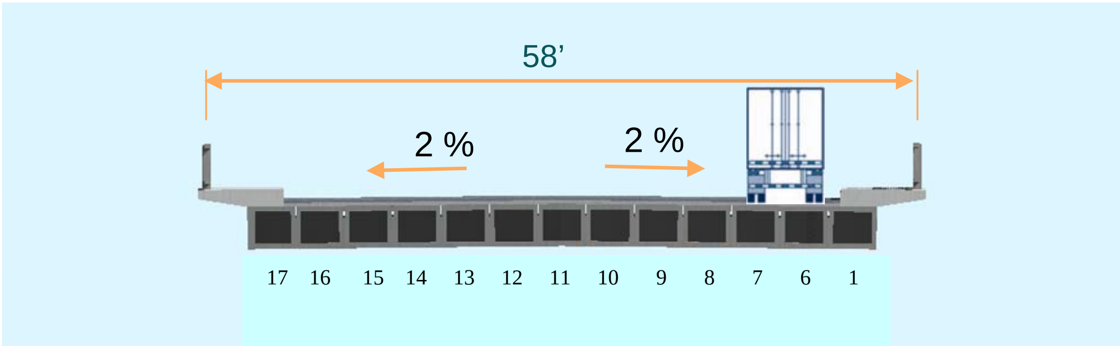

Figure 9.2 – Identification of bearing pads

Table 9.2 – Bearing pad reactions

Pad no.	1	6	7	8	9	10	11	12	13	14	15	16	17	CUMUL
Load Case	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)	P (kN)
D	-356	-328	-307	-300	-295	-292	-292	-292	-295	-299	-307	-328	-356	-4047
DSWMED	-33	-22	-14	-10	-8	-7	-7	-7	-8	-10	-14	-22	-33	-195
L625CE3	-25	-23	-22	-24	-25	-22	-19	-15	-13	-10	-7	-5	-1	-211
L625CE4	-22	-22	-23	-27	-29	-27	-22	-16	-12	-9	-7	-4	-1	-220
L625DR3	-125	-98	-72	-54	-39	-27	-20	-14	-10	-4	3	13	27	-420
L625DR4	-128	-105	-79	-58	-40	-27	-19	-13	-9	-4	2	11	23	-448
LHSCE	-28	-27	-27	-30	-32	-29	-25	-19	-15	-12	-8	-5	-1	-258
LHSDR	-74	-60	-44	-33	-24	-16	-12	-8	-5	-2	1	6	13	-258
LSIDEW	-25	-18	-12	-9	-7	-6	-6	-6	-7	-9	-12	-18	-25	-161

9.3.3 Summary of Stress Resultants

The stress resultant results are provided at a series of elements from the cantilever slab located at the face of the inclined wall, and approximately "d" away from the face. These elements are shown in Figure 9.3 and the stress resultants are presented in Table 9.3.

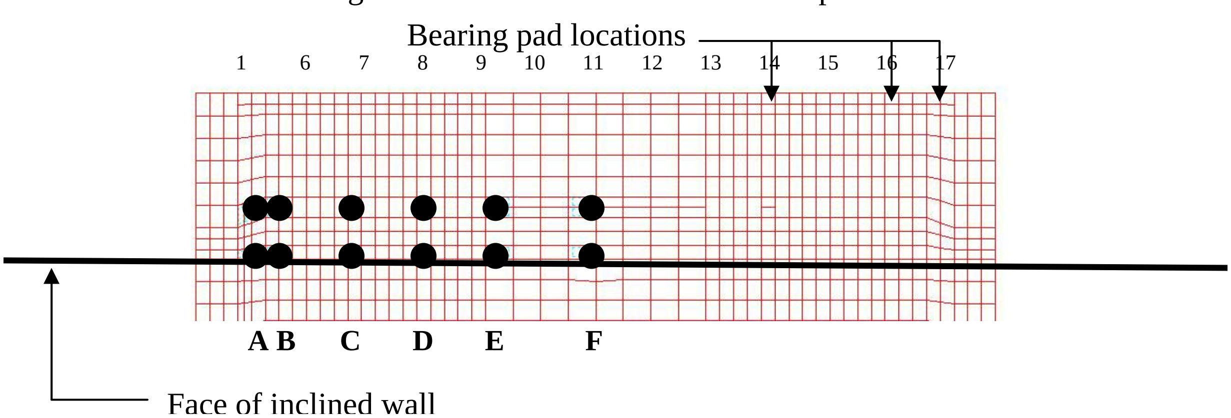

Figure 9.3 – Location of stress resultants in cantilever slab – de Blois

Figure 9.4 illustrates the distribution of the principal bending moments (per unit width of the slab) measured along the longitudinal axis of the bridge (parallel to the traffic and parallel to the direction of the main reinforcing steel in the cantilever). It should be noted that the critical location for bending, which should govern the design of the top steel, is at the face of the inclined wall.

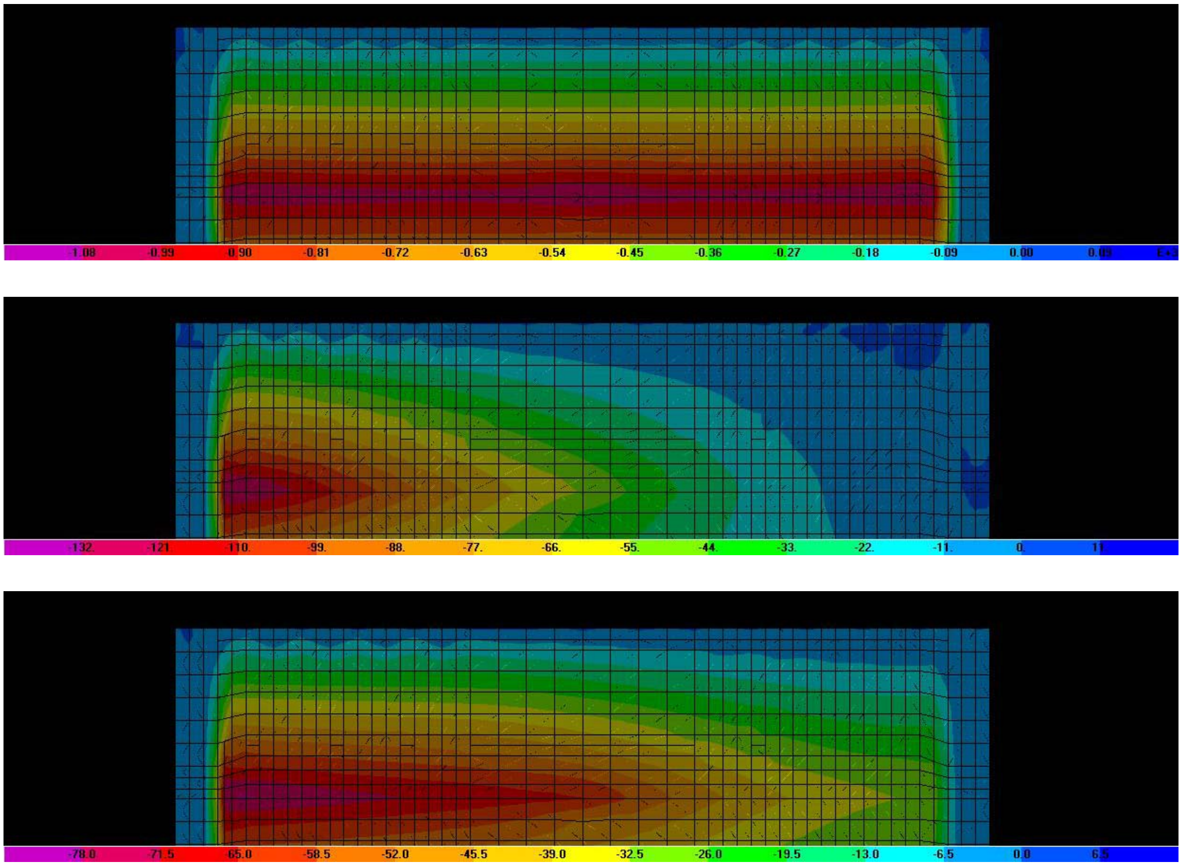

Figure 9.4 – Distribution of principal bending moment (unit (kN-m/m)
(top: dead load; middle: H20-S16 in outer lane, South side; lower: H20-S16 in centre lane, South side)

Figure 9.5 illustrates the distribution of the maximum shearing force (per unit width of the slab). It should be noted that the shearing force is critical a distance “d” away from the face of the inclined wall, not at the face of the wall, because of the presence of high compressive loads at the face. We note that “d” is the depth of the slab, measured from the centre of gravity of the # 14 top steel to the bottom face of the slab).

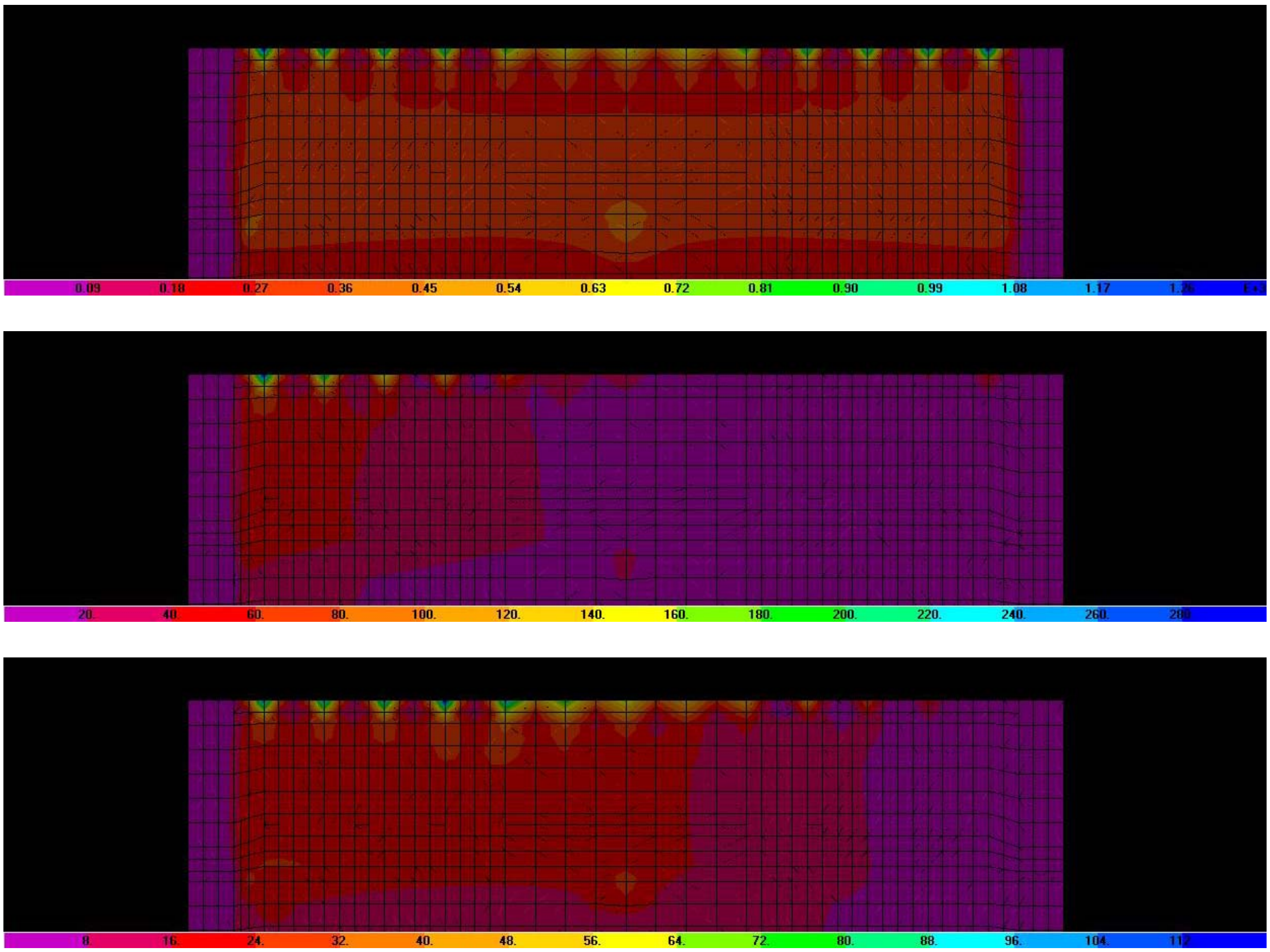

Figure 9.5 – Distribution of shear force (unit (kN/m)
(top: dead load; middle: H20-S16 in outer lane, South side; lower: H20-S16 in centre lane, South side)

Table 9.3 – Summary of stress resultants in selected elements
of the cantilever slab – De Blois

			Stress resultants at face of inclined wall		Stress resultants at "d" from face of wall	
Element	Distance from bridge centre line	OutputCase	M11 max	Vmax	M11 max	Vmax
	feet	Text	KN-m/m	KN/m	KN-m/m	KN/m
F	0 to -2	D	1164	449	726	328
		DSWMED	220	135	109	77
		L625CE3	56	19	37	17
		L625CE4	62	23	39	19
		L625DR3	100	27	66	25
		L625DR4	107	28	70	26
		LHSCE	69	24	46	21
		LHSDR	61	16	40	15
		LSIDEW	32	8	21	6
E	-6 to -8	D	1109	346	730	326
		DSWMED	229	108	117	79
		L625CE3	58	18	45	19
		L625CE4	64	24	47	22
		L625DR3	128	39	124	44
		L625DR4	137	43	133	48
		LHSCE	71	22	56	24
		LHSDR	78	24	76	27
		LSIDEW	32	7	26	8
D	-12 to -13	D	1103	353	727	335
		DSWMED	235	113	122	88
		L625CE3	61	19	45	19
		L625CE4	66	24	46	23
		L625DR3	160	52	141	56
		L625DR4	171	58	150	63
		LHSCE	74	23	54	23
		LHSDR	97	32	86	34
		LSIDEW	37	10	29	10
C	.-17 to -18	D	1118	364	733	344
		DSWMED	229	114	120	93
		L625CE3	64	20	45	19
		L625CE4	67	21	46	21
		L625DR3	192	67	154	69
		L625DR4	207	76	162	72
		LHSCE	77	23	54	24
		LHSDR	117	41	93	42
		LSIDEW	43	14	33	14
B	-23 to -24	D	1168	388	743	350
		DSWMED	208	113	108	91
		L625CE3	70	22	46	20

			Stress resultants at face of inclined wall		Stress resultants at "d" from face of wall	
Element	Distance from bridge centre line	OutputCase	M11 max	Vmax	M11 max	Vmax
	feet	Text	KN-m/m	KN/m	KN-m/m	KN/m
		L625CE4	71	21	47	19
		L625DR3	234	93	156	83
		L625DR4	251	103	164	86
		LHSCE	83	25	55	23
		LHSDR	141	56	94	50
		LSIDEW	55	25	34	21
A	.-25 to -26	D	1139	557	951	386
		DSWMED	192	189	150	103
		L625CE3	69	30	58	22
		L625CE4	70	27	59	21
		L625DR3	231	135	199	95
		L625DR4	247	149	210	102
		LHSCE	82	33	69	26
		LHSDR	140	81	120	57
		LSIDEW	55	42	45	25

9.4 Comparisons of factored resistances with factored loads – De Blois

The factored loads obtained from the 3D finite element analysis are compared with the factored resistances using the load factors (see Section 2) and the general shear design method from CSA S6 2006. Because the results from the 3D analysis are more representative of the actual behaviour of the bridge, the results from this analysis were used in assessing the factored loads on the cantilever.

Table 9.4 summarizes the factored moments and shears in the more highly stress region of the cantilever near the SE corner (near pads 1, 6 and 7). In calculating the factored shear resistance, V_r, and the flexural resistance, M_r, the procedures described in CSA S6-2006 were used. For the calculations of V_r, the General Method for shear design of Section 8.9.3.7 was used. This method accounts for the "size effect" in shear, for the interaction between shear and moment and the influence of the amount of flexural reinforcement. In addition, the vertical component of the inclined compression parallel to the bottom face of the cantilever was considered. For these calculations, the maximum moments and shears per metre width were factored with the appropriate load factors to get the factored load effects. The factored moment and shear resistances were also determined for a one metre width. The maximum factored moment at the face of the inclined wall and the factored shear and co-existing factored moment at a distance of d from the face of the wall were used.

Table 9.4 – Comparison of factored loads and factored resistances – De Blois

Element/ Location	**M_f** kNm/m	**M_r** kNm/m	**M_r/M_f**	**V_f** kN/m	**V_r** kN/m	**V_r/V_f**	**Comments**
A at face							
CL625	2014	2753	1.37				OK
H20-S16	1831	2753	1.50				OK
B at face							
CL625	2057	2753	1.34				OK
H20-S16	1871	2753	1.47				OK
C at face							
CL625	1887	2753	1.46				OK
H20-S16	1734	2753	1.59				OK
A at d							
CL625	1688			723	600	0.83	NG
H20-S16	1537			645	622	0.96	Say OK
B at d							
CL625	1317			642	643	1.00	OK
H20-S16	1201			578	664	1.15	OK

The flexural resistance is adequate for both the CL625 and H20-S16 loadings at the face of the wall support. There are regions at the edges of the cantilever that require minimum shear reinforcement. The minimum reinforcement required is the same as that required for the de la Concorde Bridge, that is, 20M single-legged stirrups on a grid spacing of 600 mm.